US Army Corps
of Engineers ®
Portland District

Salmon Recovery through John Day Reservoir

John Day Drawdown Phase I Study

Engineering Technical Appendix Slope Stability, Shoreline Erosion and Sedimentation Section






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Section 1. Introduction

This technical appendix section documents the results of the geologic evaluation for the John Day Drawdown Phase I Study. This Phase I Study is a reconnaissance-level evaluation of the potential consequences and benefits of the proposed drawdown of the John Day Reservoir. This technical appendix section supplements the main report, which describes more fully the alternatives, purpose, scope, objectives, assumptions, and constraints of the study.

Section 2. Background of the Project

In 1991, the National Marine Fisheries Service (NMFS) proposed that Snake River wild sockeye, spring/summer chinook, and fall chinook salmon be granted "endangered" or "threatened" status under provisions of the Endangered Species Act. Natural resource agencies believe that the drawdown of the 76-mile John Day Reservoir may provide substantial improvements in migration and rearing conditions for juveniles by increasing river velocity, reducing water temperature and dissolved gas, and restoring riverine habitat. It is also speculated that drawdown may improve spawning conditions for adult fall chinook by restoring spawning habitat and the natural flow regimes needed for successful incubation and emergence.

As a result, the NMFS Reasonable and Prudent Alternative Action #5 of its' Biological Opinion on Operation of the Federal Columbia River Power System (FCRPS), and subsequent reports recommended that USACE investigate the feasibility of lowering John Day Reservoir. In compliance with appropriation conditions, only two alternatives were to be evaluated: reduction of the current water surface elevation 265[1] to the level of the spillway crest that would vary between elevations 217 and 230, or reduction to natural river level elevation 165. Both alternatives were proposed by NMFS. These two alternatives were then expanded to consider each alternative with 500,000 acre-feet of flood storage and without such storage. Flood storage and hydropower are the current approved authorizations for the John Day project.

Section 3. Description of the Study Area

The Columbia River originates in Canada and flows for 300 miles through eastern Washington to Oregon and continues west to the Pacific Ocean, as shown in Figure 1. The adjoining region is mostly open country, with widely scattered population centers. The climate of the region is semiarid. Agriculture, open space, and large farms are prevalent. Lands adjacent to the reservoir are used to grow grains and other crops. The reach of the Columbia River under consideration in this report extends from John Day Lock and Dam at river mile (RM) 215.6, to McNary Lock and Dam RM 291. The body of water impounded by John Day Dam, Lake Umatilla, is referred to as the John Day Reservoir throughout this report. The John Day is the second longest reservoir on the Columbia River, extending 76 miles upstream to McNary Dam.

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[1] All elevations referred to in this Phase I Study are referenced in feet to the National Geodetic Vertical Datum.

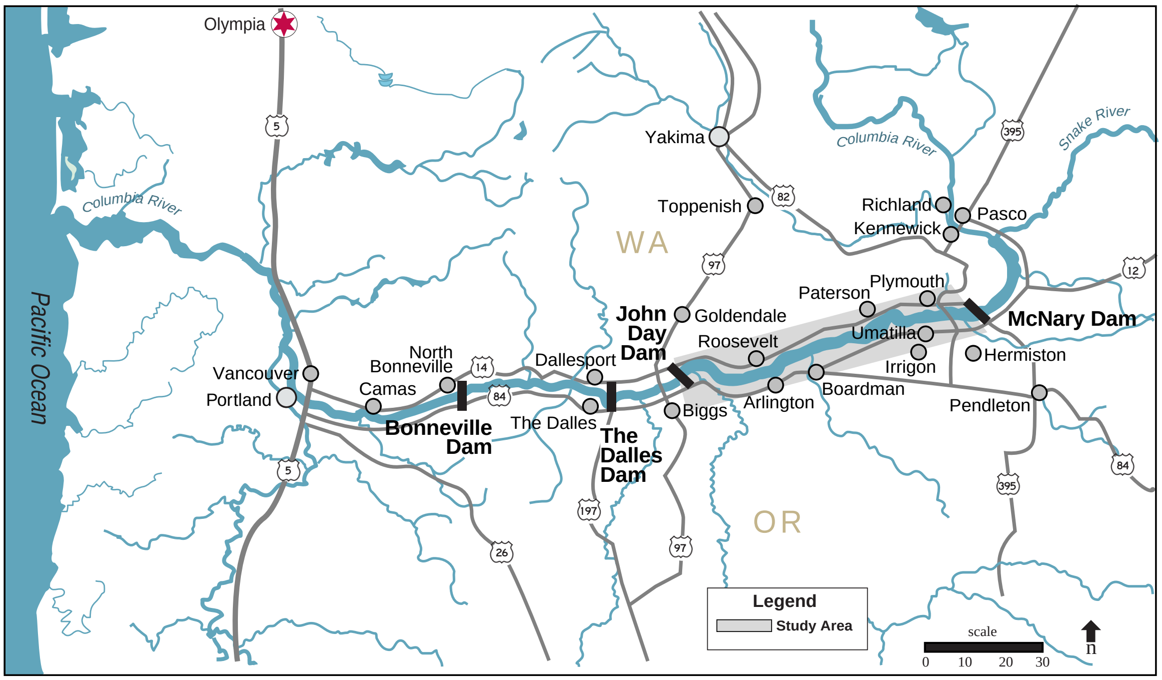


Figure 1. John Day Drawdown Phase 1 Study Area

John Day Dam and Reservoir are part of the Columbia-Snake Inland Waterway. This shallow-draft navigation channel extends 465 miles from the Pacific Ocean at the mouth of the Columbia River to Lewiston, Idaho. The entire channel consists of three segments. The first is the 40-foot-deep water channel for ocean-going vessels that extends for 106 miles from the ocean to Vancouver, Washington. The second is a shallow-draft barge channel that extends from Vancouver to The Dalles, Oregon. Although this section is authorized for dredging to a depth of 27 feet, it is currently maintained at 17 feet. The third section of the channel is authorized and maintained at a depth of 14 feet and extends from The Dalles to Lewiston. In addition to the main navigation channel, channels are dredged to numerous ports and harbors along the river.

The middle Columbia River area is served by a well-developed regional transportation system consisting of highways, railroads, and navigation channels. Railroads and highways parallel the northern and southern shores of the reservoir. Interstate 84 (I-84), a divided multilane highway, runs parallel on the south shore with the Columbia River from Portland, Oregon, to points east. Washington State Route 14 (SR-14) also parallels the Columbia River from Vancouver to McNary Dam on the north shore. Umatilla Bridge at RM 290.5, downstream from McNary Dam, is the only highway bridge linking Oregon and Washington across the Columbia River in the John Day Reservoir.

The study area includes lands directly adjacent to the reservoir as well as those directly and indirectly influenced by the hydrology of the reservoir (e.g., irrigated lands). It includes the reservoir behind the John Day Dam, and adjoining backwaters, embayments, pools, and rivers.

Section 4. Alternatives

The Phase 1 Study includes a preliminary evaluation of the impacts of the drawdown scenarios relative to the “without project condition,” which is defined as the condition that would prevail into the future in the absence of any new federal action at John Day. The four alternatives are summarized below. One of the most important constraints on the alternatives is the requirement to pass fish for river flows up to the 10-year flood flow of 515,000 cfs. Under the four alternatives, John Day Reservoir would be drawn down at a rate of one foot per day. For greater detail, please refer to the main report, *John Day Drawdown Phase 1 Study*, and *John Day Drawdown Phase 1 Study, Engineering Technical Appendix, Structural Alternatives Section*.

4.1. Spillway Drawdown without Flood Control (Alternative 1)

The first drawdown alternative is based on requirements for improved downstream fish passage conditions during both low and flood flow conditions on the Columbia River. The existing 20-bay spillway will be operated differently from current operations, but without any structural modifications. All project inflows will be directly passed through the dam spillway with the spillway gates fully opened in free overflow condition, resulting in a pool elevation that will vary from elevation 217 to 230. Impacts downstream from John Day Dam were not studied.

4.2 Spillway Drawdown with Flood Control (Alternative 2)

The second study alternative is based on requirements for improved downstream fish passage conditions during low flow periods, while maintaining authorized flood control for the John Day Project. The existing 20-bay spillway will be operated differently from current operations, but without any structural modifications. During low flow periods, project inflows will be directly passed through the dam spillway with the spillway gates set in fully open, free overflow condition. During a flood event, however, the spillway gates will be controlled to reduce downstream flood flows based on using 500,000 acre-feet of allocated project storage space. Ponding will occur upstream from the dam. Impacts downstream from John Day Dam were not studied.

4.3 Natural River Drawdown without Flood Control (Alternative 3)

The third study alternative is based on a natural river drawdown for fish passage "without flood control" condition. Natural river conditions pertain to an opening at the John Day Dam that permits acceptable upstream fish passage conditions. The size of the total dam opening must conform to two criteria based on an invert elevation at the dam of 135. The first criterion is that the opening must be sufficiently large to meet maximum allowable stream velocity criteria for sustained swim speed for the weakest salmon species, which is estimated to be 10 feet per second (fps). The second criterion is that fish passage for this opening must correspond to the 10-year annual flood peak (515,000 cfs). This alternative will require extensive modifications to John Day Dam even beyond modification of the 1,228-foot long spillway structure. Impacts downstream from John Day Dam were not studied.

4.4 Natural River Drawdown with Flood Control (Alternative 4)

This fourth study alternative is based on natural river conditions for fish passage and includes the "with flood control" condition. It requires natural fish passage conditions for both upstream and downstream directions at the dam and includes a requirement for full authorized flood control. The calculated width of the total dam opening will correspond to that previously calculated for natural river conditions without flood control (Alternative 3). Impacts downstream from John Day Dam were not studied.

Section 5. Overview

This section summarizes the results of a preliminary assessment of the impacts related to slope stability, shoreline erosion, and sedimentation from drawdown of the John Day pool. The purpose of this analysis was to determine the potential impacts associated with initial drawdown to spillway crest and to natural river with subsequent water level fluctuations from future flood events. The analysis also determined the methods associated with erosion protection for shorelines and railroad/highway embankments. Protection of the railroad/highway system is critical for continued commerce and transportation in the region due to the significant impacts to river navigation, as discussed in the *Engineering Technical Appendix, Navigation Analysis Section*. Large-scale landslides are also a potential threat to the transportation system and are evaluated in this section, but no related cost estimate was made. Additional impacts and discussion of shoreline erosion with respect to bridge and culvert structures crossing railroad/highway embankments may be found in *Engineering Technical Appendix, Shoreline Impact Evaluation Section*. Sedimentation within John Day pool since reservoir filling was assessed to determine the approximate amount and distribution of sedimentation. The impacts of sedimentation are discussed, with additional impacts and discussions presented in the *Engineering Technical Appendix - Water Quality, Sediment Quality, and Tributary Sedimentation Evaluation sections*.

Section 6. Existing Studies and Reports

6.1. Background

Many design memorandums present information about the design of the John Day Dam, and the relocation of railroads, highways, recreational facilities, and towns. Related studies include drawdown evaluations to different operating levels and cultural resource surveys.

6.2. Existing Project Information

A list of the project reports and design memoranda is included in the attachments. These reports include information about the regional geology, geologic/construction issues, railroad/highway relocation design, recreational facilities relocation design, and construction material sources. Existing topographic contour maps prepared by the U.S. Army Corps of Engineers (USACE) include 1935 bathymetric soundings of the John Day River and 1955 topography of the reservoir area above natural river level. Both of these map sources were digitized and compiled into a Geographic Information System (GIS) during this study phase. Cultural resources have been previously mapped and are part of the project GIS database, as well as the 1994 hydrosurvey data collected by USACE.

6.3. Related Studies

Reservoir shoreline stability has been analyzed in terms of geomorphic conditions by the Environmental Laboratory at Waterways Experiment Station (WES) under the Environmental and Water Quality Operational Studies (EQOS) Program. WES also participated in the System Operation Review (SOR) studies conducted from 1992 to 1996 in the Columbia River Basin. As a result of the EQOS focus, a number of technical reports have been produced dealing with identification, evaluation, and modification of reservoir shoreline stability problems. Geomorphic analysis techniques (developed by Earthquake Engineering and Geosciences Division of the Geotechnical Laboratory at WES during the SOR) address the effects of operational pool levels on cultural resources, as well as shoreline stability, at the John Day Dam and at John Day pool. The WES investigators prepared a report under the SOR study using a GIS-based geomorphic analysis procedure addressing effects of reservoir operation at a storage project (Dworshak on the Clearwater River in Idaho (Corcoran and Lawson, 1996)), and a run-of-river project, John Day. Unfortunately, funding limitations prevented completion of a detailed analysis of John Day pool.

The System Configuration Study (SCS) presented information on shoreline stability, and addressed several sites of large-scale mass movements in John Day Reservoir. This study focused on previously defined zones of mass movement near Alderdale, Washington, and Arlington, Oregon. The objective of the study was to develop historical and 1996 baseline georeferenced videographic records of mass movement at these locations. In addition, Terra Cognita at the Oregon State University Geosciences Laboratory was contracted to investigate the relationship between pool activity and mass movement. There was no formal report of the findings, which were inconclusive at the end of the study. The findings were to be formally reported as part of the SCS, however the study was stopped before the report was written. It should be noted that the reservoir stages addressed in the study did not include the substantially lower reservoir levels to spillway crest and natural river that are part of this study, but some of the information is still applicable.

A cultural resource survey and evaluation report (Wilde, Dalan, Wilke, Keuler, and Foss, 1983) includes information about the geologic history, regional geology, structural geology, and geomorphology of parcels in the John Day Reservoir in relation to mapped cultural resources.

A 1992 geotechnical investigative report of potential slope stability and erosion problems (Gustafson, 1992) prepared by a consultant for USACE, Portland District, identified the major areas of concern with respect to slope stability and shoreline erosion in the reservoir area during drawdown to minimum operating pool level (MOP). A thesis prepared by Anderson, 1971, titled "Stability of Slopes in Clay Shales Interbedded with Columbia River Basalt" for USACE, Walla Walla District analyzed areas where failures and reactivation of slides occurred during relocation construction of the highways and railroads for the John Day project.

Section 7. Study Methodology

7.1. Slope Stability and Shoreline Erosion

Literature related to reservoir slope stability and shoreline erosion was researched and reviewed. Reservoir shoreline and railroad/highway embankments were examined and evaluated by traversing John Day pool by boat and reviewing color infrared aerial photographs (Corps, July 1995), digital orthophotography (Corps, July 1994), and hydrosurvey maps (Corps, 1994) in an attempt to map and quantify those areas that would require repair and/or protection during drawdown and subsequent operation at the study alternative levels. Drawdown events from both initial drawdown and future flood events were assumed to cause failure of portions of the railroad/highway system due to rapid dewatering of the embankment. The cost associated with these rapid dewatering events were based on costs prepared by Walla Walla District for the Lower Snake River Juvenile Salmon Migration Feasibility Study. The Snake River study estimated the cost for repairs from both initial and future rapid dewatering events to both railroad and road embankments located on one side of the river. The distance is approximately the same as the length of the John Day pool. The costs estimated for the John Day Drawdown Study were increased to include repair of potential damages to railroad/highway embankments located on both sides of the Columbia River. These costs are presented in the *Engineering Technical Appendix, Engineering Cost Estimate Section*.

Costs were also prepared for placing riprap to protect railroad/highway embankments and shoreline from erosion, undercutting, and potential failure caused by river flows adjacent to the shoreline, wave action, and seasonal water level fluctuations. Protection for a full range of water levels up and down the slope is required due to the significant seasonal variations in Columbia River discharge. Riprap quantities were based on the following assumptions. The distance along the shoreline was determined from the reconnaissance work by boat. The slope length was assumed to extend from the top of proposed riprap protection on a one vertical to three horizontal theoretical slope to five feet below minimum water level (50,000 cfs) for the study alternatives. The assumed one vertical to three horizontal slope was based on the premise that slopes more shallow than this would not require riprap erosion protection. The top of proposed riprap protection is the base of existing riprap at elevation 252 feet for study alternatives 1, 2, and 3, which is either at or below the approximate water surface level from a 1 percent chance exceedance flood event (100-year frequency event). The top of rip rap for study alternative 4 (natural river without flood control) was located at five feet above the water surface elevation from a one percent chance exceedance flood event, since this alternative had significantly lower water levels than the other three alternatives. The size/thickness of riprap was based on existing riprap sizes as measured and observed in the field, calculations presented in John Day project design memorandums, the *Engineering Technical Appendix, Flood Control Evaluation Section*, and calculations for a wind generated wave. The riprap thickness used in quantity calculations and cost estimates for this study was 30 inches, a Class IV riprap where 75 percent of the material would range from 400 to 1,600 pounds and 30 percent would exceed 800 pounds. Future studies should include a detailed evaluation and analysis of protective riprap placement to further refine the size and thickness requirements and to better define where protection is required and the estimated quantities/costs.

7.2. Sedimentation

Sediment transported into the reservoir since John Day Dam was constructed was quantified by using computer models to compare pre-project and post-project surfaces. The difference between the two surfaces approximates roughly the amount of sediment in-fill or sedimentation. This is an approximate quantity and should be further refined. A detailed description of the sedimentation analysis is provided below.

Section 8. Regional Geology and History

The John Day project spans the upper reach of the Lower Columbia and to a great extent, marks the geomorphic and geo-historical boundary between the middle and lower river. The ancestral Columbia was, in geomorphic terms, an antecedent stream, meaning that the river existed in the same channel prior to the rise of the landforms that currently surround it. As the Columbia Basalt Plain rose after the end of the Ice Age, the river continued to downcut, creating an incised channel. This progressive channel incision was greatly accelerated during the waning stages of the Ice Age when the repeated breakouts of glacial Lake Bonneville sent torrential floods down the Columbia. Flooding from glacial lakes developed the bed and banklines of the Columbia and many of its major tributaries through prolonged erosion. The alluvial floodplains of the present-day river have masked the banklines in many reaches, including the 30-mile reach at the upper end of John Day Reservoir.

Section 9. Reservoir Geologic Setting

Geologic conditions are well described and documented in design memorandums produced for the John Day project (see Section 13). In general, this portion of the Columbia Plateau consists of basalt flows with an apparent dip of ½ to one degree eastward from a northwest/southeast anticlinal axis parallel and near the lower John Day River (DM 7.18). This results in higher units becoming gradually lower in elevation eastward through the reservoir area. In addition, the basalt flows have a slight apparent dip to the north and meet the Columbia Hills anticline abruptly immediately north of the Columbia River. Most of the reservoir area is in the Umatilla Basin portion of the Columbia Plateau. Bedrock units appear flat lying, from a practical standpoint, in any given local area. Contact zones between basalt flows often contain thin soil horizons and flow breccia material. The Umatilla Basin flow layering also contains two thick beds of claystone or clay shale (derived from ash deposits), commonly referred to as the upper and lower claystones. The natural dip of bedrock units and landsliding off the canyon edge have brought this claystone and slide debris down to construction elevations on both sides of the Columbia River. Construction of the highway and railroad relocations had to deal with old slide planes within the slide debris and some slide areas were reacitvated. Other types of soil materials involved in the construction work were silt, sand, gravel, and talus. Most of the fills were constructed of talus, shot rock, and gravel.

Section 10. Analysis of Reservoir Shoreline Erosion, Sedimentation and Erosion, and Slope Stability

Various studies document that reservoir shoreline erosion, sedimentation, and slope stability effects are active over a broad area, known as a zone of influence. At the John Day pool, the zone of influence extends from the pre-dam river channel where sediment is depositing below water at the toe of the slope. Higher up on the slope, to the low water line where subaerial or wind generated erosion is active, the surface of the slope is reasonably stable. Continuing upward from the low water line, at variable distances farther upslope where there is direct influence of the pool, the slope is in some stage of incremental failure. Away from the high water line and the upper edge of the vertical scarp on the slope, lies the zone of indirect influence. In this zone there are several forms of slope instability which are triggered by the loss of support when the stabilizing influence of the bank materials riverward of this zone are reduced by erosion. The zone of direct influence around John Day Reservoir has been enlarged since the flood of 1997 when peak flow was approximately 600K cfs and the pool level remained above normal for over a month (see *Engineering Technical Appendix, Flood Control Evaluation Section*). Extensive erosion damage occurred, especially in places where banklines are composed of non-cohesive soils. The literature search revealed no studies that addressed (1) enlargement of the zone of influence, (2) the condition of the influent streams, or (3) the effects on these streams of the prolonged lower reservoir operational levels. These topics will need to be evaluated in future studies.

10.1. Shoreline Erosion

10.1.1 Shoreline

For purposes of this discussion on stability and erosion, the shoreline can be generally divided into four types:

- Benches cut in bedrock with the water against bedrock
- Embankments with the water against riprap
- Natural soil with sufficient distance for beach formation
- Placed material with gravel protection or self-armoring characteristics

The Oregon shore from John Day Dam to Arlington is mostly embankment fill, protected by riprap. The Washington shore from John Day Dam to Roosevelt alternates between fill protected by riprap and exposed bedrock surfaces. The Oregon shore from Arlington to Willow Creek varies between riprap protected fills and natural or placed material. The Washington shore from Roosevelt to Crow Butte consists of riprap protected fill and natural or placed material. The Oregon shore from Willow Creek to McNary Dam is generally gently sloping natural materials. The Washington shore from slightly east of Crow Butte to McNary Dam varies from protected fills to gently sloping natural materials (see Plates 7 and 8). This is a generalized description of the reservoir shoreline and variations within these reaches.

10.1.2 Impacts

Fluctuating pool levels are known to be the primary cause of shoreline erosion in reservoirs. Normally, shoreline erosion in run-of-river reservoirs is confined to a relatively narrow band, and in the John Day Reservoir, is limited to a pool impingement zone of about 11 vertical feet. High banks composed of granular soils with low cohesion such as are found in the upper 30 miles of the reservoir, have a higher vertical wave-cut bench. The presence of this high bank is due to undercutting of the inherently unstable material, which then allows the overlying upper slope to fail from lack of support at the waterline. Upper bank stability between Crow Butte and McNary Dam has been adversely affected by the abnormally high pool stages reached during the peak of the 1997 spring runoff. There is less evidence of widespread bank instability between John Day Dam and Crow Butte on the Washington side and Willow Creek on the Oregon side. This may be due, in part, to the steep rocky shoreline in the reach downstream of Crow Butte and Willow Creek. However, there are soil areas within this rocky reach where bank loss estimates were reported to be as much as 10 horizontal feet measured landward from the shoreline. Shoreline erosion will also have an impact on cultural resources that have been identified in portions of the John Day Reservoir and documented in a report prepared by Wilde, Dalan, Wilke, Keuler, and Foss, 1983. Initial drawdown and future flood events may cause erosion of the reservoir slopes through the processes discussed above, resulting in exposure and/or destruction of these identified cultural resource sites, as well as those that have not been identified. Vandalism may also occur at the exposed cultural sites. Further discussion of these impacts is presented in the *Cultural/Tribal Resources Technical Appendix*.

10.2. Sedimentation and Erosion

As mentioned above, an attempt was made to quantify the amount of sedimentation that has occurred in the reservoir by creating pre- and post-project topographic surfaces. These surfaces were compared to estimate the amount and location of sediment erosion and accumulation since the construction of John Day Dam. The attachments illustrate the areas where sediment erosion and deposition have occurred (see plates 1-6).

10.2.1 Pre-Project 3-D Model Description

Twenty-six map sheets (dated January to March 1935) were acquired from Walla Walla District Corps of Engineers. The map scales were 1:2,000 and 1:4,000. The bathymetry on these sheets was generally crossline in nature, with spacing averaging 250 feet. Crossline surveys are acquired by navigating the survey vessel perpendicular to river flow, turning up or downstream at the end of each crossline, then continuing the survey. The spacing of individual survey points along the crossline varies, but averages approximately 40 feet. The 1935 survey covered the natural river area, with the exception of then-existing islands. The bathymetry was digitized.

To complete the pre-project 3-D model, thirty 1955 topographic map sheets were acquired from Walla Walla District. These maps covered the area from the natural river to well beyond the current maximum pool extent. Then-existing islands were also mapped. Map scale was 1:7200 with a 10-foot contour interval. The contours were digitized up to an elevation of 300 feet. Spot elevations were also digitized. To produce the 3-D model of the pre-project condition, the 1935 bathymetric data and the 1955 topographic data were merged into a single TIN (triangulated irregular network) representing the ground surface using an Arc/Info based GIS. Before accomplishing this, the 1935 data had to be referenced to the same vertical datum as the 1955 and the 1994 data, which is NGVD. The 1935 data, however, was not referenced to a constant datum, but was only adjusted for varying river flows during the survey period. Since the goal was to compare 1994 data to pre-project data, the 1935 data had to be converted from depths to NGVD. This was done by developing an additional TIN that represented the 1935 river surface profile. The profile data was available on the 1935 hydrosurvey sheets as low-water surface elevations. These elevations were located at irregular intervals, with additional elevations marked at significant changes in the river profile, such as at the head and foot of rapids. A data layer was produced depicting these elevations as lines crossing the river perpendicular to flow. The lines were attributed with the elevations listed on the map sheets. The resultant data layer was used to create a TIN. It should be noted that the TIN representing the 1935 river surface profile is very generalized in nature. This introduces the greatest potential for error in the analysis. The TINs were transformed into GRIDs for further processing. This is required when arithmetic functions are performed between surfaces. The 1935 bathymetry (as depths) was subtracted from the 1935 water surface profile to produce a GRID depicting the 1935 river morphology as NGVD elevations. The 1935 grid was appended with a grid representing the 1955 topographic contours, resulting in a grid depicting the pre-project condition. Areas where there was no data were not included in the pre-project surface or subsequent analyses (see paragraph 10.2.3).

10.2.2 Post-Project 3-D Model Description

The post-project river morphology is based on extensive hydrosurveys done in 1994. The surveys covered the entire project, including all backwater areas, such as Paterson Slough. The majority of the river was surveyed using the crossline method, with spacing of the crosslines at approximately 500 feet. The individual survey points along the crosslines average approximately 50 feet. Irregular points were collected in very shallow areas by a smaller survey vessel. All 1994 survey data was referenced as depths below the Columbia River Datum, which for the John Day pool is 257 feet NGVD. The data was converted to NGVD elevations.

10.2.3 Sedimentation Analyses

To perform the sedimentation analyses the pre-project grid surfaces (1935 and 1955 data) were subtracted from the 1994 surface by using Arc/Info software. The 1935 data covered only the area of the main channel of the natural river and did not include side channels and other areas where water was present. These areas were not included in the sedimentation analyses because there was no data available and the modeling process would yield false results with respect to erosion and deposition. Additionally, the 1955 data density is significantly less than that of the 1935 and 1994 data (reference paragraphs 10.2.2 and 10.2.3). An analysis using the pre-project surface, including the 1955 data and the 1994 surface would yield results limited to the accuracy of the 1955 data. For example, a 7-inch variation in the ground elevations would result in a quantity increase or decrease of approximately 50 million cubic yards over the length of the reservoir. The accuracy of the 1955 contours is not known, but is estimated to be 5± feet, resulting in an even larger variation in sedimentation calculations than given in the example above. Due to the accuracy limits of the 1955 data, a sedimentation analysis was made using only the 1935 and 1994 data in an attempt to quantify the amount of erosion and deposition within the main channel of the natural river, since that was the area covered by the 1935 data. The two data sets (1935 and 1994) were comparable in data density and accuracy, with 1935 data covering approximately 36 percent of the reservoir area. Results of the analysis indicates that the amount of sediment eroded from the natural river channel is approximately 22.7 million cubic yards while the amount deposited is about 78.3 million cubic yards, a net deposition of 55.6 million cubic yards. Another analysis was made using both the 1935 and 1955 data, given the 1955 data accuracy limit, to determine the amount of erosion and deposition. The coverage area is approximately 86.6 percent of the reservoir. Results indicate that 222.7 mcy of material was eroded while 176.9 mcy of sediment was deposited with a net erosion of 45.8 mcy. The values calculated are not highly accurate but provide general information about erosion and deposition. Plates 1 - 6 show the distribution of erosion and deposition in the area covered by the 1935 bathymetry and the 1955 topography based on these analyses, as well as the areas not included in the analyses. If the areas without data coverage are assumed to be areas where deposition occurred, because they are/were low, backwater areas where deposition most likely would have occurred, then the amount of deposition can be increased by the area represented by the no data areas, 13.4 percent. This results in an increase of 23.7 mcy in deposition. If the amount of deposition is increased to 200.6 mcy from 176.9 mcy, then the net erosion is decreased to 22.1 mcy. The amount of sediment that currently moves through the Columbia River system on an annual basis is 2± mcy, according to the Integrated Feasibility Report for Channel Improvements and Environmental Impact Statement, Columbia and Lower Willamette River Federal Navigation Channel report prepared by USACE, Portland District, August 1999. The report states, "Based on observed concentrations and appropriate flow-duration curves, USACE estimated that the average annual suspended sediment yield at Vancouver has been reduced from 12 mcy per year before any dams were built, to only 2 mcy per year under today's conditions." The quantity calculated for net erosion from John Day Reservoir, 22.1 mcy, covers about a 59-year period, with most of the erosion probably occurring since 1968 when John Day pool was filled. This is equivalent to about 0.37± million cubic yards of sediment passing John Day Dam per year over a 59-year period and about 0.85± mcy over a 26-year period. Both of these values are within the standard volume of material moving through the entire system on an annual basis. Some of the material deposited in the natural river channel would have been derived from the reservoir slopes and other previously dry areas that were inundated during initial reservoir filling. Landslides may be an additional source of material (see Slope Stability paragraph below). If the analyses discussed above are reviewed in a general sense, the majority of erosion appears to have been from around the islands and reservoir slopes in the upper third of the reservoir. Erosion of these previously dry areas was probably the result of wind generated waves during and after reservoir filling. The upper third of the reservoir contains alluvial sands and silts and sand dune islands that are easily eroded. Future studies should verify the validity of these calculations with respect to river hydrology and particle size through sediment sampling and testing programs. Additional research should be conducted in an attempt to locate survey maps that provide complete coverage of the reservoir area prior to John Day Dam construction. The missing data between 1935 and 1955 is important for quantifying the amount of sedimentation that would create turbidity impacts from a drawdown, as discussed below and in the *Engineering Technical Appendix, Water Quality Section*. Analyses should be performed with any additional survey data to more accurately determine the overall sedimentation and distribution within the reservoir.

10.2.4 Impacts

The upper reach of the reservoir, which has incised into late Holocene glacio-fluvial deposits of immense volume and expanse, is expected to undergo a substantial change as a result of John Day pool drawdown. The greatest impact will be turbidity caused by the influx of this sediment into the system from initial drawdown, increased water velocity, wind generated wave impingement, and subsequent flood events. Contributing to the turbidity will be sediment washed from the reservoir sideslopes and eroded from the river channel, where significant amounts of sediment deposition have been identified, as described above. The tributary streams to John Day pool and the alluvial fans associated with the mouths of the Umatilla River, Willow Creek, Rock Creek, and the John Day River (see *Engineering Technical Appendix, Sedimentation Evaluation Section*) will also contribute significantly to the system turbidity. Some minor tributaries are expected to also contribute substantial volumes of fine-grained silts and clays, increasing reservoir or river turbidity on a seasonal basis. These minor tributaries include Glade Creek, Alder Creek, Wood Gulch, Pine Creek, and Blaylock Canyon. A related impact from the influx of this sediment into the river system will be the deposition of the material downstream of the John Day project. An additional impact from drawdown will be the increase in blowing sand and dust. The currently submerged sediment deposited on the reservoir sideslopes, as well as the tributary alluvial fans mentioned above, will be exposed after drawdown and become a significant source for blowing material. Revegetation of these areas will help control the amount of blowing material, but this will be difficult to do in such a dry climate.

10.3. Slope Stability

Typically, the instability of a reservoir slope is confined to that portion of the slope that is subject to wave impingement. Currently for the John Day Reservoir, the instability is limited to the drawdown zone, or about 11 feet in elevation, and between 0 and 30 feet of slope length (depending upon the nature and condition of the materials comprising the slope and the underlying strata). Additional drawdown will increase the exposure of the reservoir slopes to wave action, thereby expanding the potential for further slope destabilization. Railroad and highway embankments located on those slopes are also at risk of failure not only from erosion, undercutting, and loss of underlying support but also from rapid dewatering of the embankment material.

Mass movement of slopes is another slope stability issue that may be affected by reservoir drawdown. Previously identified areas of movement or potential movement were reviewed, evaluated, and documented in the geotechnical reconnaissance report of potential slope stability and erosion problems prepared in 1992 and a thesis prepared in 1971. Field reconnaissance techniques employed during the 1992 geotechnical investigation to look for potentially unstable sites during future lower pools consisted of examining the cuts, fills, riprap zones, natural slopes, and constructed slopes for cracks, flaws, disturbances, erosion, and failures. These could indicate conditions that might develop into problems during lower pool levels. A thesis prepared by Anderson, 1971, titled "Stability of Slopes in Clay Shales Interbedded with Columbia River Basalt" for the U.S. Army Corps, Walla Walla District analyzed areas where failures and reactivation of slides occurred during relocation construction of the highways and railroads for the John Day project. The study looked at the soil and geologic properties of the clay-shale interbeds that are present within the basaltic bedrock and is exposed along the valley walls. Stability of the slopes was determined by the presence and orientation of fractures within the clay shales and the inclination of the underlying basalt flows. The results of the study indicate that the reservoir includes a 15-mile zone of landslide topography associated with clay-shale interbeds that are susceptible to failure, if erosional downcutting were to remove enough material to initiate progressive failure.

Stability problems may also occur with currently submerged fine-grained alluvial slopes located at the mouths of tributaries during drawdown and subsequent flood events. These areas include Umatilla River, Willow Creek, Rock Creek, and John Day River. Failure of the unconsolidated, saturated sediment slopes may occur from rapid dewatering, similar to the railroad and highway embankments.

10.3.1 Landslide Analysis

Features in the John Day Reservoir that are expected to be impacted by a drawdown include unstable colluvial reservoir slopes, pre-existing zones of fractured clay shale associated with a decomposed volcanic ash, overlain by basalt, and sheared zones associated with major, but ancient, tectonic movements. The greatest risk of slope failure during reservoir drawdown lies in the basalt deposits overlying weak clay zones of decomposed volcanic ash. According to Anderson (1971), the decomposed volcanic ash, or clay shales with oriented fractures or fissures that are present in bedding with an inclination approaching or exceeding the residual angle of internal friction of the clay shale are the most unstable. Of the 12 failures that occurred during relocation construction of the highways and railroads, 6 were on the Oregon side and 6 on the Washington side. A number of the failed slopes on both sides of the river reactivated and failed again after repair. The reason for slide reactivation was unknown. There is evidence of prolonged creep-type movement in these same areas, particularly in the vicinity of Alder Creek on the north shore and Willow Creek on the south side of the river, according to the 1992 geotechnical investigation, where landslide areas were evaluated that could potentially experience movement during reservoir drawdown to MOP.

10.3.1.1. Oregon Side of Reservoir

During construction relocation of the highways and railroads along the John Day Reservoir, six slope failures occurred (See Plates 7 and 8). The "Murphy" slide located between I-84 milepost 143 and 146, six miles east of Arlington, was reactivated about 10 months after the initial failure, eight months after completion of repair work. The failure was a slide within a slide debris area that was unloaded and buttressed during construction. No sign of stress was observed during the 1992 investigation. The "Mainline" slide located at milepost 140.6, 2 miles east of Arlington, was also reactivated six months after initial failure, one month after repair work was completed. The "Tieline" slide located near milepost 140.3 on a tieline between an old and new track failed at three different times at three different location. "Heppner Junction Slide #1" occurred along the eastbound lane of I-84 at milepost 145.8, while "Heppner Junction Slide #2" was located in the same area at milepost 145.2. The "Highway" slide was also located next to the eastbound lane at milepost 139. Most of the repairs involved unloading the top of the active zone and placing a berm of river gravels to load and buttress the toe. The angle of inclination of the clay shale interbeds was about eight degrees in the failed slopes.

10.3.1.2. Washington Side of Reservoir

Six slope failures also occurred along the Washington shore during relocation work of the highway and railroad (See Plates 7 and 8). Slide #1 located near milepost 147.6 near Alderdale was reactivated two months after initial failure and less than one month after repair operations. Slide #2 was located at milepost 147.2, slide #3 at milepost 141.9, slide #4 at milepost 142.2, and slide #5 at milepost 150.4. Slide #6 located at milepost 149.3 was reactivated five years after initial failure and repair. Slide #1 just downstream of Alderdale near milepost 147.6 was instrumented with "Kelly wire" extensometers during pool raise because there were cracks between the highway and the railroad (personal communication Gustafson/McDevitt 30 June 1992). During the 1992 investigation, the highway and adjacent bin wall showed visual dip, and 100 to 150 linear feet of ground cracking was observed between the highway and railroad (Gustafson, 1992). The railroad track and grade appeared to have been repaired in this area. The crack on the riverside of the highway appeared to be the result of translational sliding rather than slumping where the lower portion would raise up. It was not apparent if the movement was on a deep slide plane or shallow surface movement. The angle of inclination of the clay shale interbeds ranged from 8 to 14 degrees in the failed slopes. According to Anderson (1971) the clay shale on the Oregon shore has a higher moisture content than the clay along the Washington shore. The Washington samples disintegrated upon contact with water, known as slaking, while the Oregon samples remained intact. The readiness to slake indicates, among other things, that the clay shale has the capacity to swell and swelling accompanied by forces of gravity are the major influences for the development of progressive failure on a long-term basis (Bjerrum, 1967). The information above indicates that the Washington shore is more susceptible to future instability than the Oregon side.

10.3.2 Railroad/Highway Embankment Evaluation

Generally, in the lower two-thirds of the reservoir, railroad embankments are located immediately adjacent to and sometimes through the reservoir. Railroad embankment material and protective riprap cover extends below the water surface to a depth of 5 feet below minimum pool design (elevation 257 at the dam) or one wave height, whichever is greater. The railroad foundation through this reach is most commonly on clay slide debris. The main railroad moves inland on the Oregon side from about Willow Creek eastward, with an exception in the Boardman area where the railroad moves close to the reservoir again, but this area is generally gently sloping natural materials. The railroad on the Washington side remains immediately adjacent to the reservoir up to just east of Paterson where it moves inland, with the exception of a small area midway between Paterson and Plymouth where the railroad runs adjacent to the reservoir.

10.3.3 Impacts

As mentioned above, the proposed drawdown will increase the exposure of the reservoir slopes to wave action and increase the potential for slope destabilization. Loss of the railroad/highway system is not an acceptable outcome from reservoir drawdown and subsequent water level fluctuations; however, the embankments for these features are located in a vulnerable area where wave impingement, undercutting, erosion, rapid dewatering, and ultimately failure are a likely scenario. The 1992 reservoir drawdown test of Lower Granite and Little Goose reservoirs was conducted at a rate of two feet per day. The adjacent railroad embankment experienced some movement during drawdown and resulted in track misalignment. Adjacent Federal, State, and County roads all experienced movement, cracking, slumping, piping, and failure to varying degrees. Sliding activity was observed in natural soil slopes consisting of silts, sands, and gravels. The known areas of mass movement in John Day Reservoir are also subject to either reactivation, as in the documented history of the "Murphy" slide, for example, or increased rates of movement, such as with the slide near Alderdale. In addition, the Port of Arlington has expressed concern about the slope stability of the filled area located on the riverside of the port where recently installed grain elevators are reported to have "tilted". This is probably a foundation problem, but may be impacted by a drawdown.

Section 11. Preliminary Protective Measures

No surface inspection or data study can anticipate all the hidden physical conditions, flaws, or adverse circumstances in geotechnical work, so drawdown of the reservoir to new levels should be treated somewhat like pool raising. Careful inspection and monitoring should be programmed for the reservoir perimeter during the drawdown period. Below are some additional protective measures.

11.1. Potential Landslide Area Instrumentation

The known ground cracking west of Alderdale between the highway and railroad should be instrumented before any drawdown. A survey should be conducted to establish both horizontal and vertical control points in the area and at least one slope indicator should be installed to determine the depth and type of movement, and if it occurs during or after drawdown. Consideration should be given to performing similar surveys and installing a slope indicator at the Murphy slide on the Oregon side.

11.2. Railroad/Highway Embankment Protection

Various measures can be used to protect slopes and embankments from erosion and failure, including bioremediation (vegetation), riprap, gabions, or reinforced earth. Existing slope protection along the John Day Reservoir shoreline consists almost exclusively of riprap, because of the water velocities and wave heights that present. The four alternatives proposed for John Day drawdown would each have similar or greater water velocities and wave heights, plus significant water level fluctuations, such that riprap would be the only protective measure that could provide adequate protection from erosion and failure in a cost effective manner. For railroad/highway embankments, riprap erosion protection should be placed on all slopes steeper than one vertical on three horizontal from five ft above the one percent chance exceedence flood event (100-year frequency event) to five feet below the low water operating level for each study alternate (50,000 cfs). Water surface levels for the one percent flood event for alternatives 1,2, and 4 were either at or above the existing toe of riprap placed on railroad/highway embankments to elevation 252. Riprap quantities for these alternatives were calculated using elevation 252 as the top of riprap. The top of riprap for Alternative 3 was calculated at five feet above the one percent flood event water surface levels from John Day Dam upstream to where that elevation was within 10 feet of elevation 252 and then elevation 252 was used as the top of riprap elevation. This situation only occurred at the uppermost reach of the area identified to be riprapped. Areas to be protected with riprap were roughly identified during the reconnaissance boat trip described above. These areas included areas where the existing railroad/highway embankment was immediately adjacent to or actually in the reservoir. According to USACE' Riprap Classification Chart, Class IV riprap[2] should be used based on the observations and field measurements of existing riprap that was sized based on calculations presented in the John Day design memorandums for railroad relocations (DM 7.6). Riprap size requirements were also presented in the Flood Control Evaluation appendix where Class III riprap was determined based on increased water velocities. Preliminary wave heights were also evaluated and indicated a Class IV riprap size requirement. Future studies should further evaluate the areas that require riprap erosion protection, refine the riprap size requirements, and more accurately determine the slope angle where riprap would be placed.

11.3. Sedimentation

The quantity and location of sediment in-fill was roughly calculated during this study, as described above. The size and extent of the reservoir preclude the use of any reasonably cost-effective means of controlling and preventing this loose sediment from running into the river system during drawdown. Subsequent water level fluctuations would continue the process of "washing" the sediment from the reservoir slopes and erosion of the thick deposits at the mouths of the tributaries, resulting in long-term turbidity and water quality issues. Future studies should evaluate the long-term environmental impacts this would have on the river system as well as potential impacts to the quality of water supplies, and attempt to estimate the duration of the turbidity effects.

11.4. Shoreline Erosion

The areas of greatest concern are those areas where the railroad/highway embankments are located immediately adjacent to the reservoir. Other river access features, such as docks, boat ramps, marinas, swimming beaches, ports, culverts, and bridges would be either abandoned or relocated (see *Engineering Technical Appendix - Recreation Site Impacts, Navigation Analysis, and Shoreline Impact Evaluation sections*). Similar erosion control measures as those proposed for the railroad/highway embankments could be used where unstable shoreline conditions were identified, but those areas have not been quantified for this study phase. Future studies should address this issue.

[2] 75 percent of which ranges from 400 to 1,600 pounds

Section 12. Estimated Quantities

The *Engineering Technical Appendix, Engineering Cost Estimates Section* contains a detailed description of the costs associated with riprap erosion protection. Below is a summary of the riprap quantities for the four study alternatives. Quantities are the same for the first two alternatives because the area of placement is the same. As discussed above, the top of riprap used in the quantity calculations for the first two alternatives was elevation 252 feet, which is the bottom of existing riprap. This elevation is either at or below the water surface elevation for the one percent chance exceedence flood event (100-year frequency event). The bottom elevation for riprap was also the same for the first two alternatives and represents an elevation five feet below the low water operating level (50,000 cfs). The quantity of riprap for the third alternative, natural river without flood control, is less than the other alternatives because the top of riprap was placed at five feet above the water surface for the one percent flood event. This elevation was significantly below the bottom of existing riprap and resulted in reduced quantities.

Table 1.
Riprap Quantities per Alternative

Study Alternative	Riprap Quantity (Cy)
Alt 1 – Spillway w/o Flood Control	3,371,529
Alt 2 – Spillway w/ Flood Control	3,371,529
Alt 3 – Natural River w/o Flood Control	2,635,890
Alt 4 – Natural River w/ Flood Control	5,732,465

Section 13. References

John Day Lock and Dam Design Memorandums

DM#	Title	Date/revisions
1	Hydrology	1 August 1956
		Revised 5 August 1957
2	Site Selection Report	15 June 1956
3	General Design Memorandum	23 June 1958
4	First-Step Cofferdam	1 August 1958
5	North Shore Relocation (2 Volumes)	13 May 1960
	Supplement No. 1 – Design and Cost Revisions	8 November 1961
	Supplement No. 2 – Revisions to Earthwork Design	3 January 1962
	Supplement No. 3 – Roosevelt Storage Yard and Connecting Track	9 January 1963
	Supplement No. 4 – Relocation El Paso Natural Gas Company Lines (Voided: covered in Design Memorandum No. 5.6)	1 August 1963 Revised 1 November 1963
	Supplement No. 5 – Utilities Relocations, Plymouth, Washington and Vicinity	22 September 1965
	Supplement No. 6 – Track Construction with Continuous Welded Rail	22 June 1964
	Supplement No. 7 – County Road Paterson to Plymouth	8 January 1965
5.1	SP&S Rwy., Towal to Rock Creek	28 May 1962
5.2	SP&S Rwy., Rock Creek to Sundale, Washington and State Hwy. 8, Rock Creek to Fountain	13 October 1961
5.3	SP&S Rwy. and Washington State Hwy. No. 8 Roosevelt to Pine Creek	4 December 1962
5.4	Washington State Hwy. No. 8, Carley to Whitcomb	
5.5	SP&S Rwy. and Washington State Hwy. No. 8 Pine Creek to Carley	18 November 1963
5.6	SP&S Rwy., Whitcomb to King	23 June 1965
5.7	SP&S Rwy., Carley to Whitcomb	24 June 1964

5.8	SP&S Rwy., Miller's Island to Cliffs	26 September 1962
5.9	SP&S Rwy., Cliff's to Towal	4 December 1964
5.10	SP&S Rwy., Sundale to Roosevelt	13 June 1963
5.11	Deleted	
5.12	Relocation of Portions of Washington Primary State Hwy. No. 8	24 February 1960
	Supplement No. 1 – Four O'Clock Rapids to Chapman Creek	12 June 1961
	Supplement No. 2 – Towal to Rock Creek	6 October 1961
	Supplement No. 3 – Rock Creek Culvert Repair	25 October 1963
5.13	SP&S Rwy., Track Laying and Ballast	21 July 1965
5.14	SP&S Rwy., Station Facilties	7 November 1966
5.15	SP&S Rwy., Salvage of Existing Line	
5.16	Instrumentation for SP&S Rwy. and PSH12	7 November 1966
6	North Shore Temporary Project Office and Visitor Facilities	17 September 1958
7	Relocation on Oregon Shore (2 Volumes)	15 December 1959
	Supplement No. 1 – Revision in Design and Cost Allocation	13 October 1961
	Supplement No. 2 – Earthwork Design Criteria	26 February 1962
	Supplement No. 3 – Relocation of Columbia Basin Electric Cooperative Facilities	18 March 1964
	Supplement No. 4 – Relocation of Power and Telephone Facilities	3 August 1964
		Revised 6 January 1965
	Supplement No. 5 – Protection of County Roads, River Banks, and Structures, Umatilla Area	18 November 1966
7.1	Relocation of Union Pacific Bridge, John Day River	October 1958
		Revised June 1959
7.2	Relocation Interstate Hwy. 80N and Union Pacific Railroad, Rufus to John Day River	6 June 1962
7.3	UPRR Shoofly and Hwy. 30 Detour, Dam Site Area	15 April 1959
7.4	Shoofly on UPRR and Detour in Hwy. 30 at Watchman's Dip	15 June 1959

7.5	Interstate Hwy. 80N, John Day River Bridge	30 April 1961
7.6	Relocation UPRR and Location Interstate Hwy. 80N, John Day River to Hook	23 November 1960
7.7	Deleted	
7.8	Deleted	
7.9	Interstate Hwy. 80N, Arlington Viaduct	17 January 1963
7.10	Deleted	
7.11	Deleted	
7.12	Deleted	
7.13	UPRR Bridges at Arlington and Willow Creek, Oregon	28 July 1959
7.14	UPRR and Interstate Hwy. 80N, East Watchman's Dip To Quinton	9 November 1962
7.15	UPRR and Interstate Hwy. 80N, Hook to East Watchman's Dip	7 September 1962
7.16	Deleted	
7.17	Interstate Hwy. 80N, Blalock to Arlington West	16 November 1962
7.18	UPRR, Arlington East to Willows	20 January 1964
7.19	UPRR, Blalock to Arlington West	13 July 1965
7.20	UPRR and Interstate Hwy. 80N, Arlington Area	22 April 1963
7.21	UPRR Shoofly and Interstate Hwy. 80N Detour, Blalock	29 June 1961
7.22	UPRR and Interstate Hwy. 80N, Quinton to Blalock	17 July 1964
7.23	Deleted	
7.24	Interstate Hwy. 80N and Morrow County Roads	15 January 1964 Revised 4 September 1964
7.25	UPRR, Willows to Messner	3 December 1964
7.26	Grading, Drainage, and Surfacing for UPRR, Heppner Branch Facilities, Interstate Hwy. 80N, and Oregon State Hwy. 74, Heppner Junction Area	3 February 1964
7.27	Deleted	
7.28	Instrumentation for UPRR and I80N	4 November 1966
8	Relocation of Pacific Telephone and Telegraph Company Facilities in Morrow, Gilliam, and Sherman Counties, OR	16 January 1959
9	Concrete Aggregate Investigations	14 April 1959

	Supplement No. 1 – Additional Investigation Goodnoe Terrace	11 October 1961
10	Part 1, Real Estate Dam Site Construction Area and North Shore Access Road	7 March 1958
11	Relocation of Arlington, Oregon	
	Volume 1 – Designs and Cost Estimates	19 September 1958
	Volume 2 – Real Estate	19 September 1958
	Volume 3 – Preliminary Report	July 1957
	Supp. 1 – Relocation PT&T Facilities	12 April 1960
	Supp. 2 – Relocation Foundation Treatment	9 September 1960
	Supp. 3 – Relocation of Streets and Utilities	20 October 1960
	Supp. 4 – Relocation PP&L Facilities	22 March 1961
	Supp. 5 – Relocation of Television Distribution System	
	Supp. 6 – Storm Runoff Drainage System	20 March 1963
	Supp. 7 – Air Conditioning Facilities (City Hall)	26 March 1964
	Supp. 8 – Addition to Storm Runoff System	9 September 1965
	Letter Supp. 9 – Public Parking, Arlington	25 February 1966
12	Relocation of Boardman, Oregon	10 September 1963
13	Deleted	
14	North Shore Access Road	3 March 1958
15	Preliminary Design Report, Powerplant	February 1961
15.1	Auxiliary Fishwater Supply, South Shore	June 1960
15.2	Powerhouse Station Service Power Supply	November 1960
15.3	Powerhouse Architectural Design	December 1962
15.4	Powerhouse Structural Design	March 1962
15.5	Turbines and Governors	
15.6	Powerhouse Air Conditioning Design	June 1962
15.7	Powerhouse Piping Design	March 1963
15.8	Powerhouse Mechanical Design	July 1963
15.9	Powerhouse Electrical Design	
15.10	Powerhouse Control Equipment	
16	Spillway, Navigation Lock, Right Abutment Embankment, and North Shore Fish Facilities	11 August 1959

	Supp. 1 – Design Analysis, Tainter Gate Anchorage	1 February 1960
	Supp. 2 – Navigation Lock Model Studies	18 October 1960
	Supp. 3 – Navigation Lock Sill Blocks	12 July 1963
	Supp. 4 – Spillway Gantry Crane, Stoplogs, and Lifting Beam	5 November 1965
	Supp. 5 – Extension of Lock Guide Wall "D"	3 May 1965
	Supp. 6 – Temporary Unwatering Facilities for Navigation Lock Monolith Modifications	15 October 1965
	Supp. 7 – Trans-Shipping Facilities	15 October 1965
	Supp. 8 – Navigation Lock Monolith Modification	29 June 1966
	Supp. 9 – Navigation Lock Floating Guide Wall "B"	14 January 1966
17	Exploratory Drilling and Grouting Navigation Lock	20 May 1958
18	South Non-Overflow Dam	3 August 1960
--	Cemetery Relocations, Washington Shore	15 July 1960
20	Visitor Facilities and Project Beautification	31 January 1968
20.1	Visitor Interpretive Display Facilities	
20.2	Project Service Facilities	
21	Second-Step Cofferdam	27 October 1961
22	South Shore Permanent Fish Facilities	28 January 1963
23	Superceded by 23.1	
23.1	Relocation of Boardman Public School, Boardman, Oregon	21 July 1965
24	Relocation of Arlington Elementary School	28 September 1959
25A	Preliminary Master Plan	5 March 1959
25B	Master Plan	19 October 1965
25B,C-1	Preimpoundment Tree Planting and Fencing Public Use Areas	1 August 1963
25B,C-2	Lepage Park, John Day River	18 November 1966
25.1	South Shore Public Access Facilities	18 July 1967
25.2	North Shore Public Access Facilities	21 August 1967
26	Water Supply, Storage, and Distribution	20 October 1960
27	Deleted	
28	Relocation of Municipally-Owned Property, City of Boardman, Oregon	10 September 1963

29	Relocation of Municipally-Owned Property, Arlington, Oregon	15 August 1960
30	Modification to McNary Fish Facilities	15 June 1962
31	Relocation of Roosevelt Elementary School, Klickitat County, Washington	1 November 1962
32	Deleted	
33	Deleted	
34	Foundation Grouting and Drainage	16 November 1962
35	Navigation Lock Fire Protection	25 May 1961
36	North Shore Fishway Pumphouse Crane and Trashrack Cleaning Facilities	28 March 1961
37	Deleted	
38	Protection of Umatilla, Oregon	8 August 1962
38.1	Relocation of Municipally-Owned Facilities, Umatilla, Oregon	4 March 1966
39	Deleted	
40	Plan of Relocation, Irrigon Cemetery	18 April 1962
41	Plan of Relocation, Boardman Cemetery	1 May 1963
42	Relocation of Field Office Facilities	20 September 1962
43	Cost Allocation Studies	7 August 1962
44	Reservoir Clearing	5 January 1967
45	South Shore Reservoir Access Roads	
46	Fish Hatchery	
47	Wind-Wave Investigations	

Section 14. Other Reports and Publications

Anderson Jr., Roy A., 1971, "Stability of Slopes in Clay Shales Interbedded with Columbia River Basalt," Master of Science Thesis for University of Idaho Graduate School and U.S. Army Corps of Engineers Walla Walla District, Walla Walla, WA, 297 p.

Bjerrum, L., 1967, "Progressive Failure in Slopes of Overconsolidated Plastic Clay," Journal of the Soil Mechanics and Foudation Division, A.S.C.E., Vol. 93, No. SM 5, Proceedings Paper 5456, 50 p.

Corcoran, Maureen K., Smith, Lawson M., and Nickens, Paul R., 1996, "Development of a Geomorphology Based Framework for Cultural Resources Management, Dworshak

Reservoir, Idaho," U.S. Army Corps of Engineers Waterways Experiment Station, Report GL-96, Vicksburg, MS, 57 p.

Gustafson, Lewis A., 1992, "John Day (Lake Umatilla) Project Minimum Pool Mitigation, Geotechnical Investigation of Potential Slope Stability and Erosion Problems," U.S. Army Corps of Engineers Portland District, Contract DACW57-92-M-1827, Portland, OR, 5 p.

Hodge, Edwin T., 1932, "Report of Dam Sites on Lower Columbia River," U.S. Army Corps of Engineers Pacific Division, Portland, OR, 84 p.

Hodge, Edwin T., 1938, Geology of the Lower Columbia River: Bulletin of the Geologic Society of America, Volume 49, p. 831-930.

Orr, Elizabeth L. and Orr, William N., 1996, "Geology of the Pacific Northwest," McGraw-Hill Companies, Inc., New York, NY, 409 p. U.S. Army Corps of Engineers, 1999, "Integrated Feasibility Report for Channel Improvements and Environmental Impact Statement, Columbia & Lower Willamette River Federal Navigation Channel," Volume I: Main Report and Exhibits, U.S. Army Corps of Engineers Portland District, Portland, OR, p. 5-4.

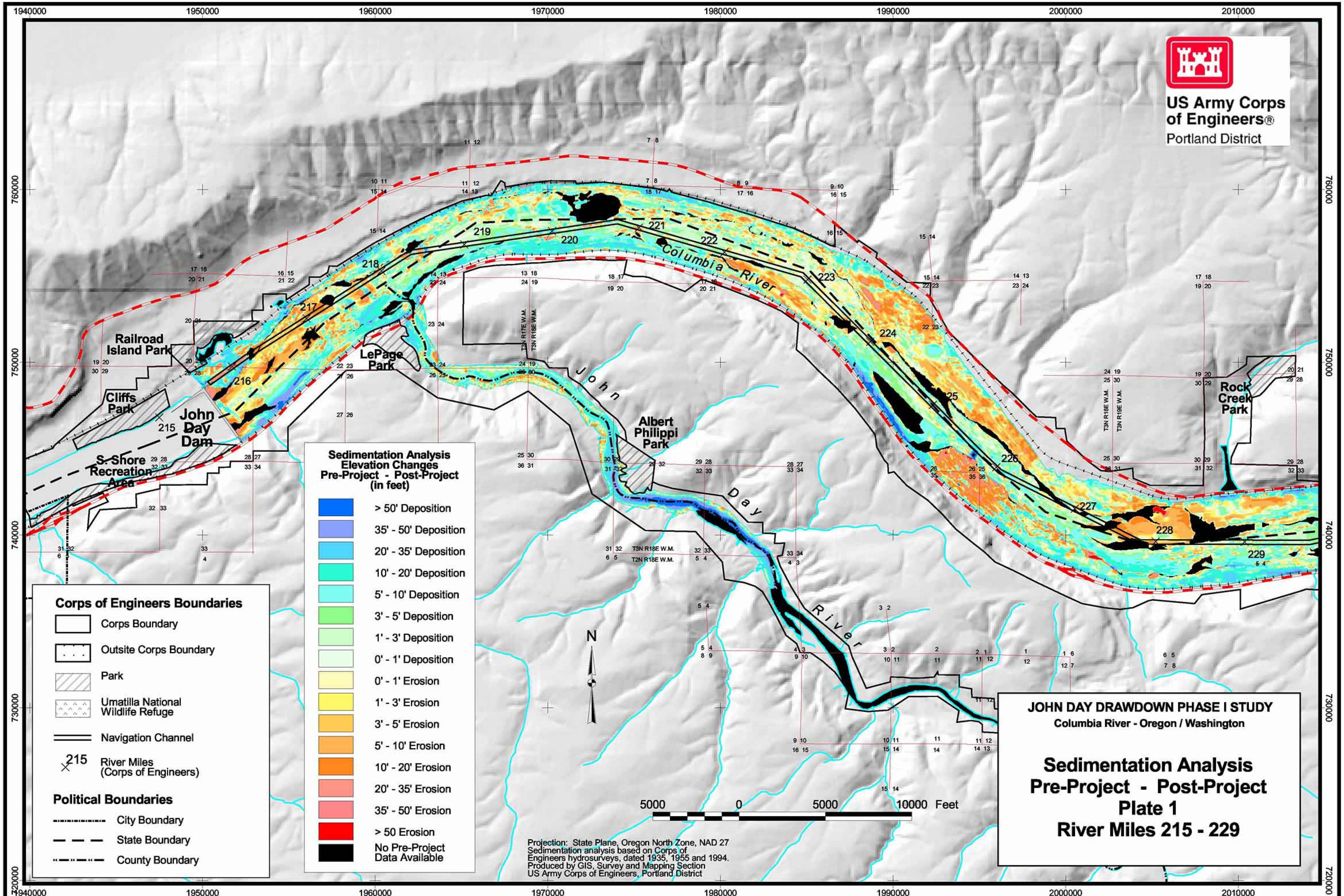
U.S. Army Corps of Engineers, 1993, "1992 Reservoir Drawdown Test Lower Granite and Little Goose Dams," U.S. Army Corps of Engineers Walla Walla District, Walla Walla, WA, 141 p.

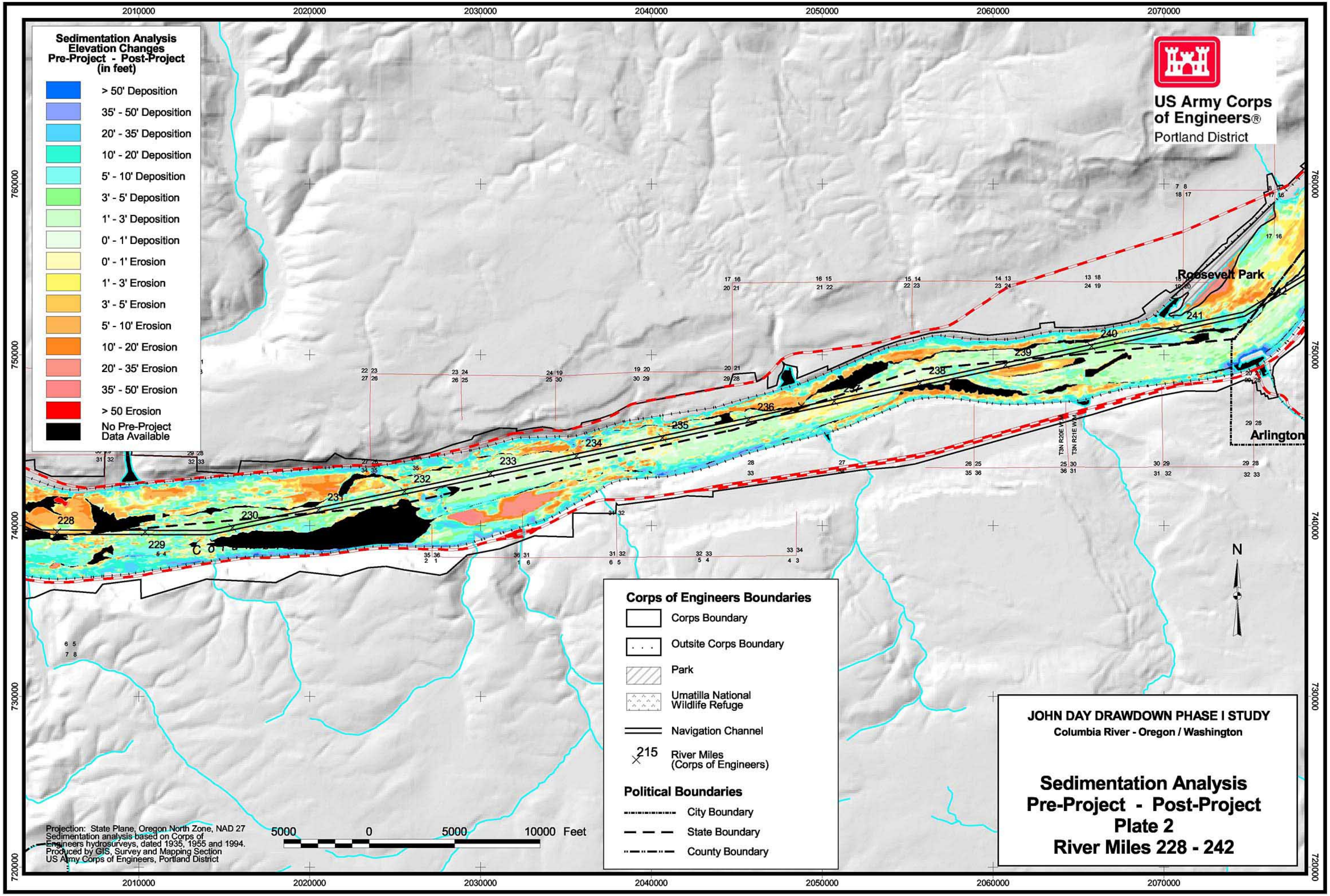
U.S. Army Corps of Engineers, 1992, "Report on Impacts and Measures for Interim Drawdown Levels of John Day pool (DRAFT)," U.S. Army Corps of Engineers Portland District, Portland, OR, 24 p.

U.S. Army Corps of Engineers, 1989, "Columbia River and Tributaries Review Study, Project Data and Operating Limits," 1989, U.S. Army Corps of Engineers North Pacific Division, Portland, OR, 291 p.

Woelke, Franziska, 1996, "Morphometric Analysis of Landslide and Slope Stability on the North Shore of the John Day Reservoir, Columbia River, Oregon and Washington," Master of Science Thesis for Oregon State University and U.S. Army Corps of Engineers Portland District, Portland, OR, 163 p.

Plates


US Army Corps of Engineers®
Portland District
Columbia River
John Day River
Railroad Island Park
Cliffs Park
John Day Dam
S. Shore Recreation Area
LePage Park
Albert Philippi Park
Rock Creek Park
Sedimentation Analysis
Elevation Changes
Pre-Project - Post-Project
(in feet)
> 50' Deposition
35' - 50' Deposition
20' - 35' Deposition
10' - 20' Deposition
5' - 10' Deposition
3' - 5' Deposition
1' - 3' Deposition
0' - 1' Deposition
0' - 1' Erosion
1' - 3' Erosion
3' - 5' Erosion
5' - 10' Erosion
10' - 20' Erosion
20' - 35' Erosion
35' - 50' Erosion
> 50 Erosion
No Pre-Project Data Available
Corps of Engineers Boundaries
Corps Boundary
Outsite Corps Boundary
Park
Umatilla National Wildlife Refuge
Navigation Channel
River Miles (Corps of Engineers)
Political Boundaries
City Boundary
State Boundary
County Boundary
N
5000 0 5000 10000 Feet
Projection: State Plane, Oregon North Zone, NAD 27
Sedimentation analysis based on Corps of Engineers hydrosurveys, dated 1935, 1955 and 1994.
Produced by GIS, Survey and Mapping Section
US Army Corps of Engineers, Portland District
JOHN DAY DRAWDOWN PHASE I STUDY
Columbia River - Oregon / Washington
Sedimentation Analysis
Pre-Project - Post-Project
Plate 1
River Miles 215 - 229



Sedimentation Analysis
Elevation Changes
Pre-Project - Post-Project
(in feet)
> 50' Deposition
35' - 50' Deposition
20' - 35' Deposition
10' - 20' Deposition
5' - 10' Deposition
3' - 5' Deposition
1' - 3' Deposition
0' - 1' Deposition
0' - 1' Erosion
1' - 3' Erosion
3' - 5' Erosion
5' - 10' Erosion
10' - 20' Erosion
20' - 35' Erosion
35' - 50' Erosion
> 50 Erosion
No Pre-Project Data Available
US Army Corps of Engineers®
Portland District
Roosevelt Park
Arlington
Corps of Engineers Boundaries
Corps Boundary
Outsite Corps Boundary
Park
Umatilla National Wildlife Refuge
Navigation Channel
215
River Miles (Corps of Engineers)
Political Boundaries
City Boundary
State Boundary
County Boundary
JOHN DAY DRAWDOWN PHASE I STUDY
Columbia River - Oregon / Washington
Sedimentation Analysis
Pre-Project - Post-Project
Plate 2
River Miles 228 - 242
Projection: State Plane, Oregon North Zone, NAD 27
Sedimentation analysis based on Corps of Engineers hydrosurveys, dated 1935, 1955 and 1994.
Produced by GIS, Survey and Mapping Section
US Army Corps of Engineers, Portland District
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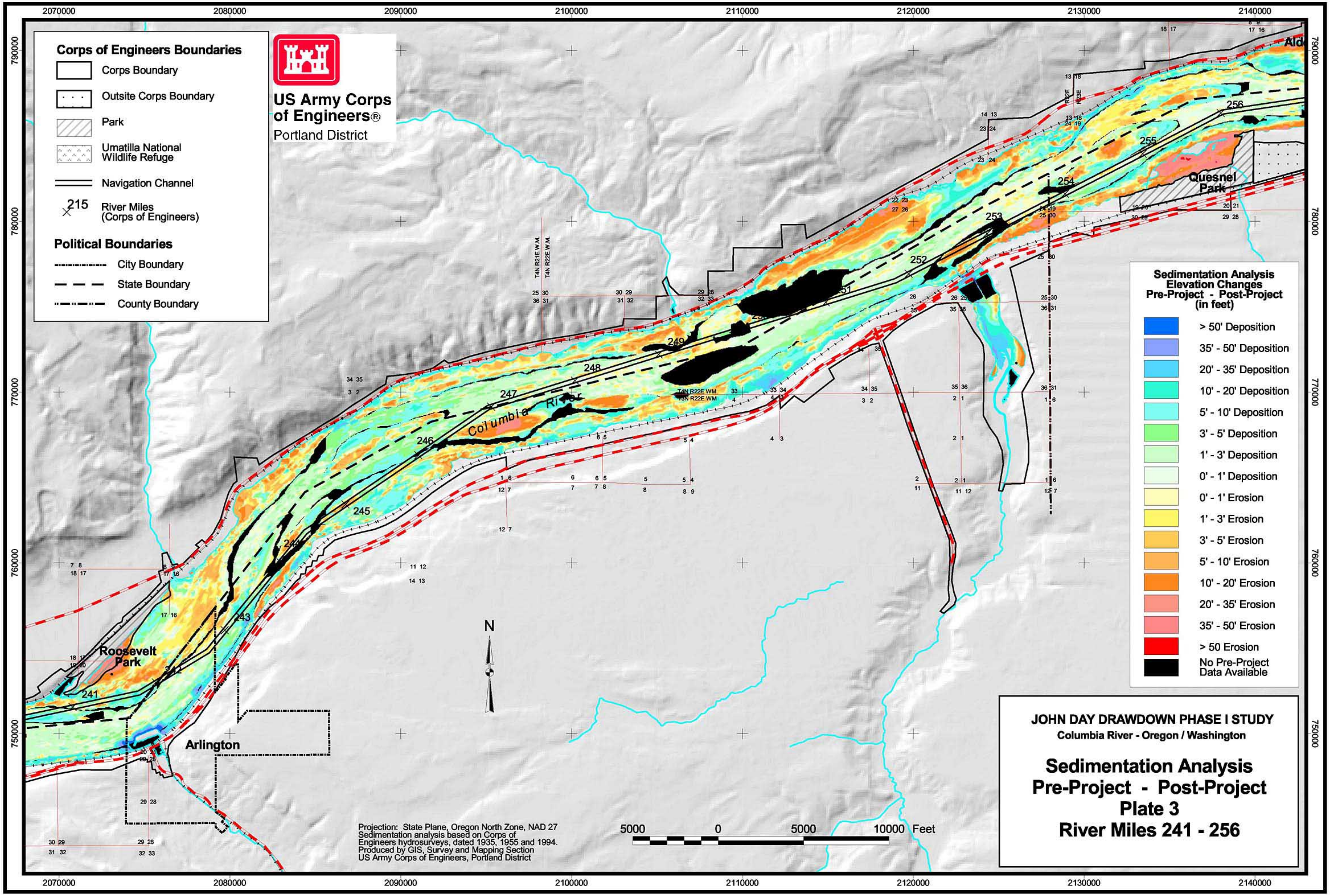
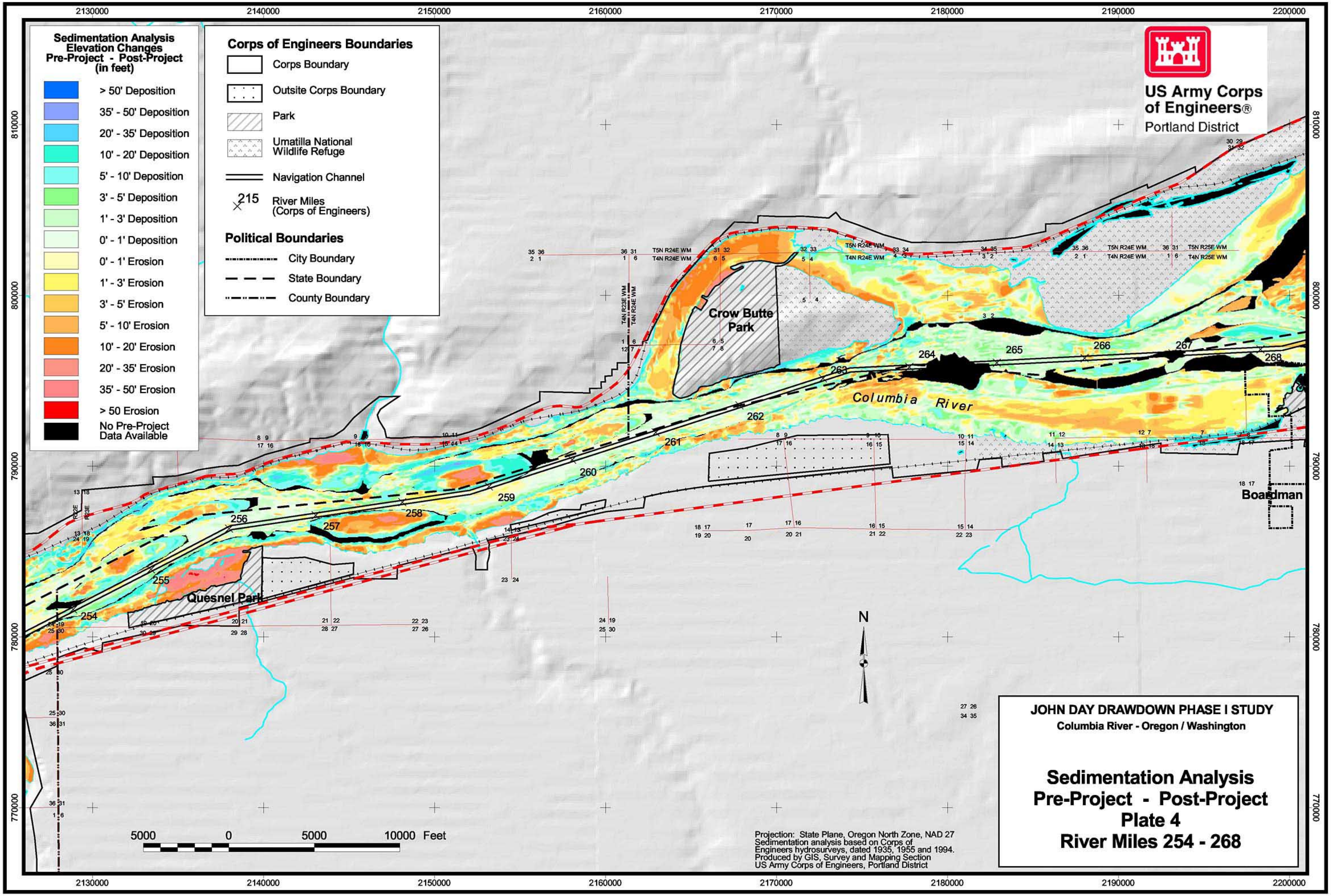
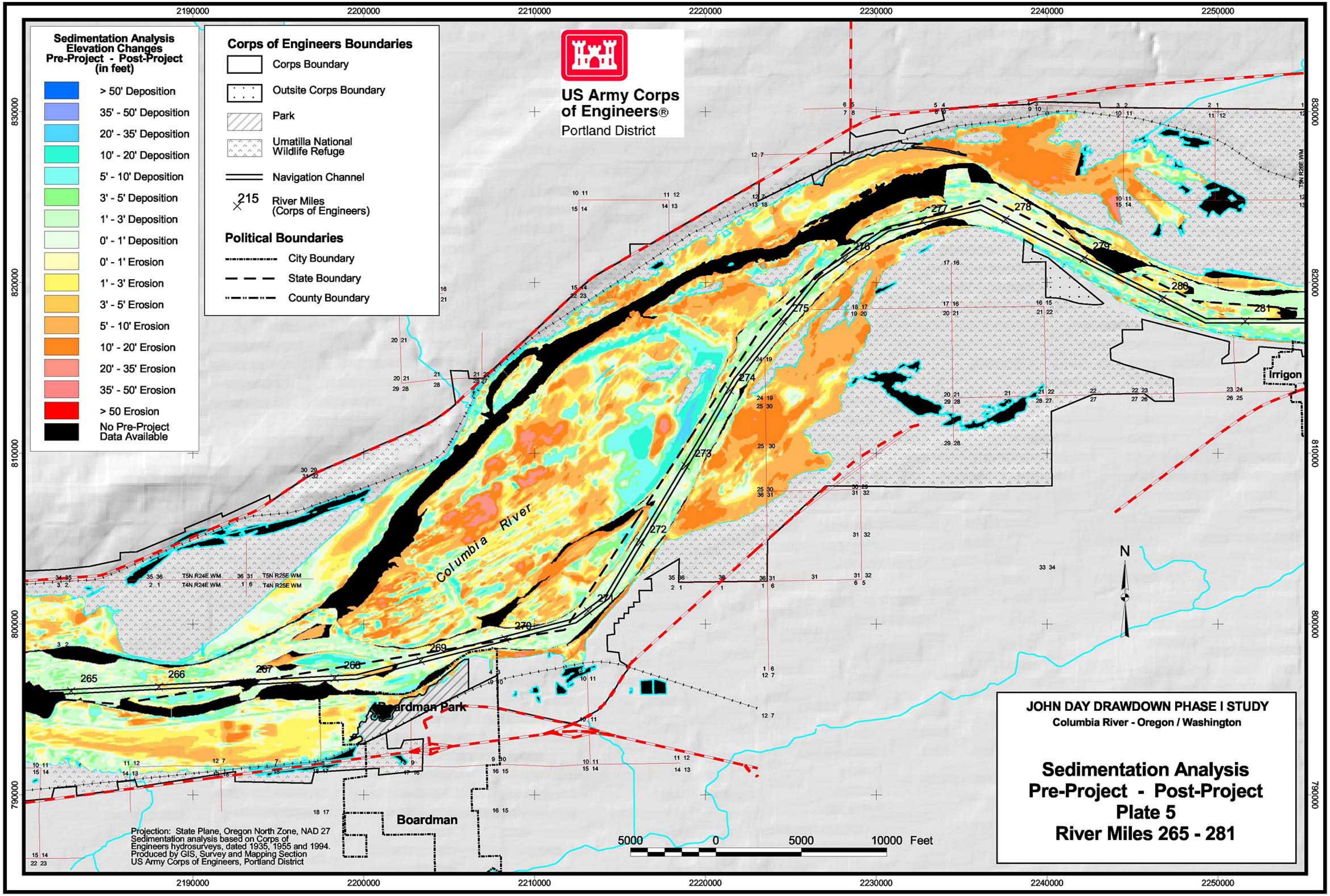
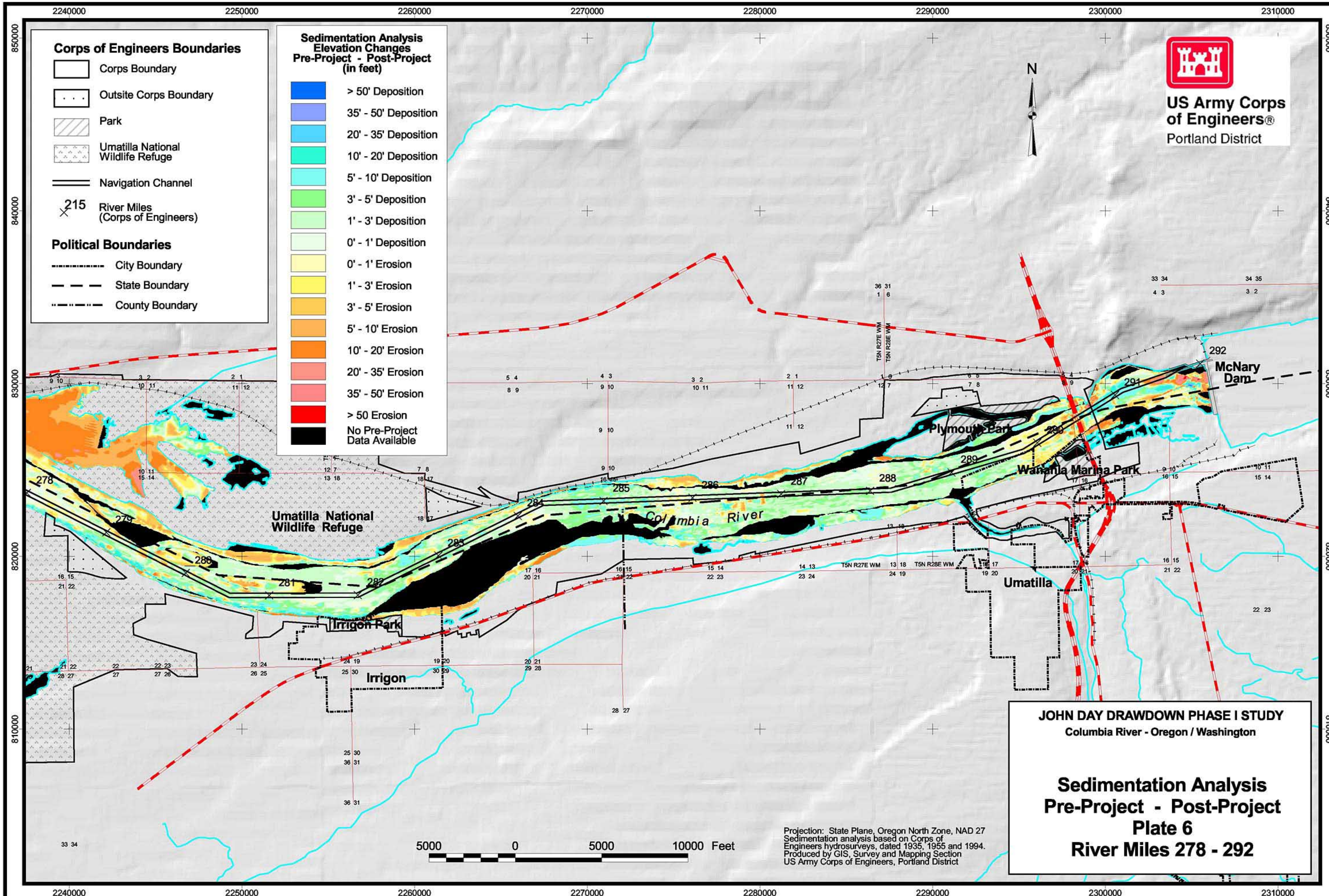


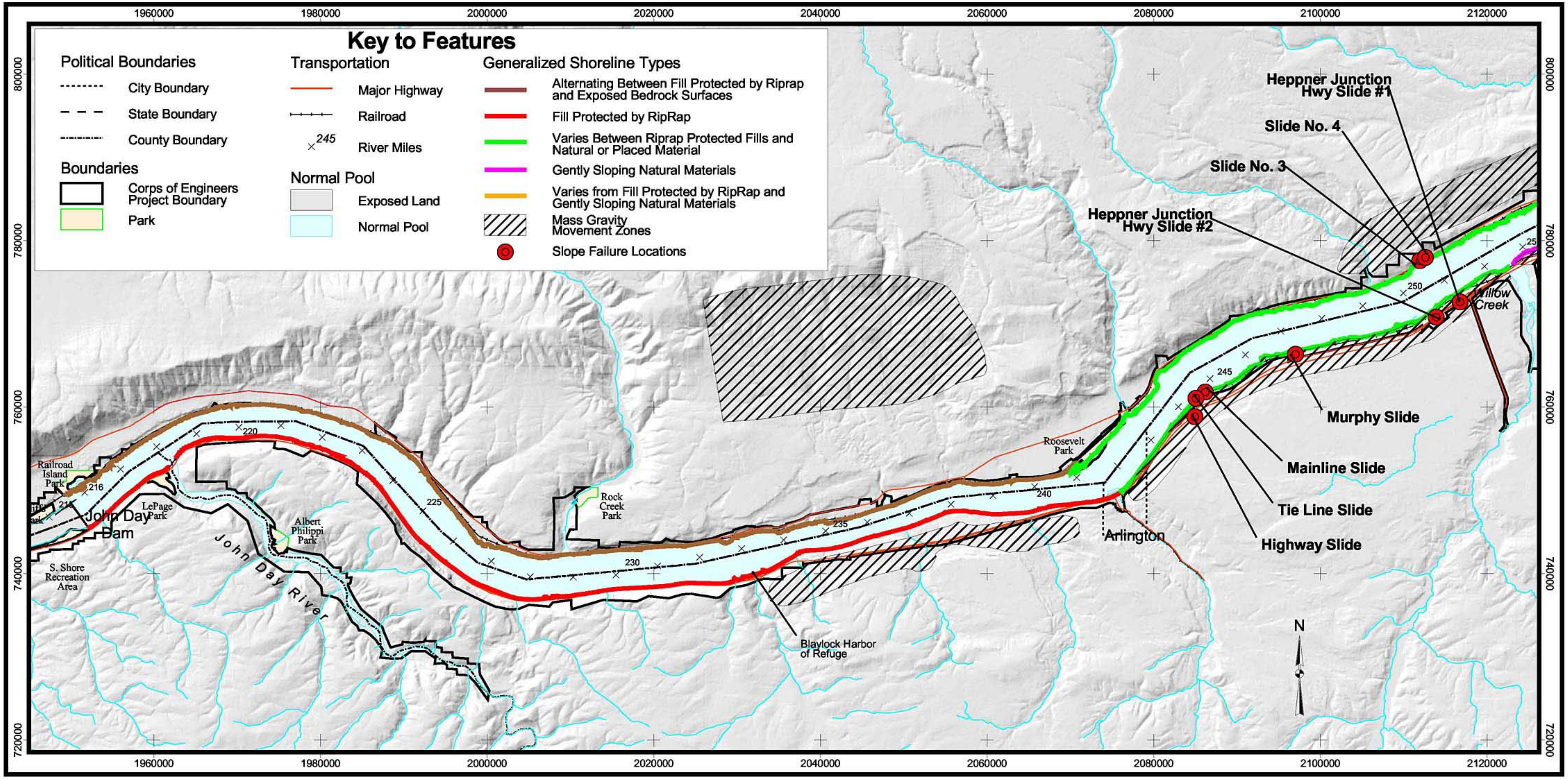
Corps of Engineers Boundaries
Corps Boundary
Outsite Corps Boundary
Park
Umatilla National Wildlife Refuge
Navigation Channel
215 River Miles (Corps of Engineers)
Political Boundaries
City Boundary
State Boundary
County Boundary
US Army Corps of Engineers®
Portland District
Sedimentation Analysis Elevation Changes Pre-Project - Post-Project (in feet)
> 50' Deposition
35' - 50' Deposition
20' - 35' Deposition
10' - 20' Deposition
5' - 10' Deposition
3' - 5' Deposition
1' - 3' Deposition
0' - 1' Deposition
0' - 1' Erosion
1' - 3' Erosion
3' - 5' Erosion
5' - 10' Erosion
10' - 20' Erosion
20' - 35' Erosion
35' - 50' Erosion
> 50 Erosion
No Pre-Project Data Available
JOHN DAY DRAWDOWN PHASE I STUDY
Columbia River - Oregon / Washington
Sedimentation Analysis
Pre-Project - Post-Project
Plate 3
River Miles 241 - 256
Projection: State Plane, Oregon North Zone, NAD 27
Sedimentation analysis based on Corps of Engineers hydrosurveys, dated 1935, 1955 and 1994.
Produced by GIS, Survey and Mapping Section
US Army Corps of Engineers, Portland District
5000 0 5000 10000 Feet
N
Columbia River
Roosevelt Park
Arlington
Quesnel Park
T4N R21E W.M.
T4N R22E W.M.
T4N R22E WM
T3N R22E WM
241
243
244
245
246
247
248
249
251
252
253
254
255
256
2070000
2080000
2090000
2100000
2110000
2120000
2130000
2140000
750000
760000
770000
780000
790000


Sedimentation Analysis Elevation Changes Pre-Project - Post-Project (in feet)
> 50' Deposition
35' - 50' Deposition
20' - 35' Deposition
10' - 20' Deposition
5' - 10' Deposition
3' - 5' Deposition
1' - 3' Deposition
0' - 1' Deposition
0' - 1' Erosion
1' - 3' Erosion
3' - 5' Erosion
5' - 10' Erosion
10' - 20' Erosion
20' - 35' Erosion
35' - 50' Erosion
> 50 Erosion
No Pre-Project Data Available
Corps of Engineers Boundaries
Corps Boundary
Outsite Corps Boundary
Park
Umatilla National Wildlife Refuge
Navigation Channel
215
River Miles (Corps of Engineers)
Political Boundaries
City Boundary
State Boundary
County Boundary
US Army Corps of Engineers®
Portland District
Crow Butte Park
Quesnel Park
Columbia River
Boardman
N
5000 0 5000 10000 Feet
Projection: State Plane, Oregon North Zone, NAD 27
Sedimentation analysis based on Corps of Engineers hydrosurveys, dated 1935, 1955 and 1994.
Produced by GIS, Survey and Mapping Section
US Army Corps of Engineers, Portland District
JOHN DAY DRAWDOWN PHASE I STUDY
Columbia River - Oregon / Washington
Sedimentation Analysis
Pre-Project - Post-Project
Plate 4
River Miles 254 - 268



Sedimentation Analysis
Elevation Changes
Pre-Project - Post-Project
(in feet)
> 50' Deposition
35' - 50' Deposition
20' - 35' Deposition
10' - 20' Deposition
5' - 10' Deposition
3' - 5' Deposition
1' - 3' Deposition
0' - 1' Deposition
0' - 1' Erosion
1' - 3' Erosion
3' - 5' Erosion
5' - 10' Erosion
10' - 20' Erosion
20' - 35' Erosion
35' - 50' Erosion
> 50 Erosion
No Pre-Project Data Available
Corps of Engineers Boundaries
Corps Boundary
Outsite Corps Boundary
Park
Umatilla National Wildlife Refuge
Navigation Channel
215 River Miles (Corps of Engineers)
Political Boundaries
City Boundary
State Boundary
County Boundary
US Army Corps of Engineers®
Portland District
Columbia River
Boardman Park
Boardman
Irrigon
N
5000 0 5000 10000 Feet
Projection: State Plane, Oregon North Zone, NAD 27
Sedimentation analysis based on Corps of Engineers hydrosurveys, dated 1935, 1955 and 1994.
Produced by GIS, Survey and Mapping Section
US Army Corps of Engineers, Portland District
JOHN DAY DRAWDOWN PHASE I STUDY
Columbia River - Oregon / Washington
Sedimentation Analysis
Pre-Project - Post-Project
Plate 5
River Miles 265 - 281


Corps of Engineers Boundaries
Corps Boundary
Outsite Corps Boundary
Park
Umatilla National Wildlife Refuge
Navigation Channel
215 River Miles (Corps of Engineers)
Political Boundaries
City Boundary
State Boundary
County Boundary
Sedimentation Analysis Elevation Changes Pre-Project - Post-Project (in feet)
> 50' Deposition
35' - 50' Deposition
20' - 35' Deposition
10' - 20' Deposition
5' - 10' Deposition
3' - 5' Deposition
1' - 3' Deposition
0' - 1' Deposition
0' - 1' Erosion
1' - 3' Erosion
3' - 5' Erosion
5' - 10' Erosion
10' - 20' Erosion
20' - 35' Erosion
35' - 50' Erosion
> 50 Erosion
No Pre-Project Data Available
US Army Corps of Engineers®
Portland District
N
Umatilla National Wildlife Refuge
Columbia River
Plymouth Park
Wanapa Marina Park
McNary Dam
Umatilla
Irrigon Park
Irrigon
278
279
280
281
282
283
284
285
286
287
288
289
290
291
292
T5N R27E WM
T5N R28E WM
2240000
2250000
2260000
2270000
2280000
2290000
2300000
2310000
850000
840000
830000
820000
810000
5000 0 5000 10000 Feet
Projection: State Plane, Oregon North Zone, NAD 27
Sedimentation analysis based on Corps of Engineers hydrosurveys, dated 1935, 1955 and 1994.
Produced by GIS, Survey and Mapping Section
US Army Corps of Engineers, Portland District
JOHN DAY DRAWDOWN PHASE I STUDY
Columbia River - Oregon / Washington
Sedimentation Analysis
Pre-Project - Post-Project
Plate 6
River Miles 278 - 292



Projection: State Plane, Oregon North Zone, NAD 27
Produced by GIS, Survey and Mapping Section
US Army Corps of Engineers, Portland District

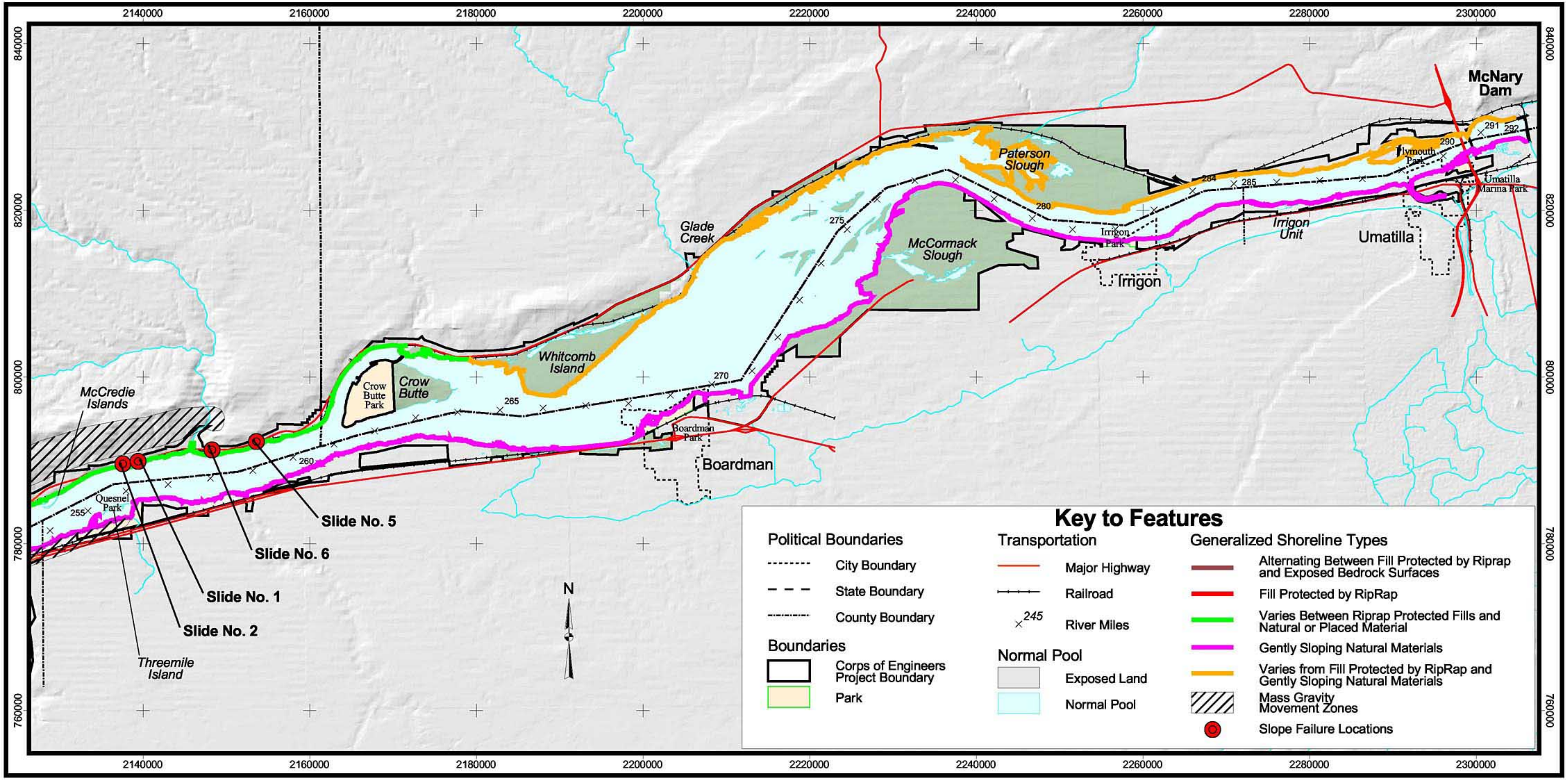
2 0 2 4 6 Miles

US Army Corps of Engineers
Portland District

JOHN DAY DRAWDOWN PHASE I STUDY
Columbia River - Oregon / Washington

Generalized Shoreline Description and Location of Slope Failures

Plate 7



Projection: State Plane, Oregon North Zone, NAD 27
Produced by GIS, Survey and Mapping Section
US Army Corps of Engineers, Portland District

2 0 2 4 6 Miles

US Army Corps of Engineers
Portland District

JOHN DAY DRAWDOWN PHASE I STUDY
Columbia River - Oregon / Washington

Generalized Shoreline Description and Location of Slope Failures

Plate 8